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EARTH OVERFLOW DIKE, JIM WOODRUFF DAM

by James M. Polatty

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James M. Polatty

I.

The Apalachicola River system with its main tributaries, the Flint and Chattahoochee, is one of the largest river systems in the southeastern United States, the drainage area above Woodruff Dam is approximately 17,150 sq. miles.

Near Chattahoochee, Florida, the Mobile District of the Corps of Engineers, is constructing the Jim Woodruff Dam. The dam or the reservoir is located in the states of Florida, Georgia and Alabama, and is the furthest downstream structure, and first to be constructed in the improvement of this river system. The Woodruff Dam is primarily for navigation and secondarily a run of river power plant. The structure itself is 6,130 feet in length and consists of a fixed crest spillway, a lock, a gated spillway, a power house, an earth switchyard and an earth dike.

The flow of the Apalachicola River at the dam site has varied between 5,120 cfs. and 273,000 cfs., however the structure itself is designed to take care of a considerably larger flood than has been recorded. To take care of this design flood, the entire structure, with the exception of the powerhouse, is designed for overtopping by flood waters. However, based on the flood data, the concrete fixed crest spillway with a crest elevation of 79 will be overtopped an average of every 27 years, the lock walls at elevation 82 once every 76 years, while the earth dike, with a crest at elevation 85 will be overtopped in something over 100 years.

The site of the dam is in the northeastern portion of the Marianna lowlands, a topograph division of the Coastal Plains province, underlain by soluble and easily degraded limestone. The Suwanee and Tampa formations are both present in the foundation of the dam, and weathering, leaching, oxidation and solution channels are in evidence throughout the area. The characteristics of these rocks range widely from a soft granular earthy limestone to a dense dolomitic limestone, both containing many clay, sand filled or open pockets, solution channels, and cavernous horizons.

II.

The earth dike, transition section and switchyard as shown in figure 1 are conventional rolled fill structures having an impervious core and upstream section, and a pervious downstream section as shown in figure 2. The impervious core in the middle third extends downward to foundation rock with a 14' width of contact. The design and construction of the earth work were and are in accordance with the current general procedures of the Corps of Engineers.

Both faces of these structures are surfaced with grouted riprap, and an asphalt roadway will be placed on top to serve as an access road and parking area to the powerhouse.

As is customary in structures on pervious foundations, the plans call for the construction of a cutoff grout curtain below the core trench and extending fifty foot into rock. The pervious conditions of the rock at this project intensified the necessity of using extreme care and the development of new techniques and economical methods and materials in order to get a satisfactory job at a minimum of costs.

This report covers construction performed under two contracts, each covered by a different set of specifications and performed by a different contractor. The specifications for the first contract were issued in 1947 and the second in 1952, representing a gap of 5 years. During this time new materials, procedures, developments, and experience resulted in changes in the requirements of the specifications.

III. Soil Work

The major changes in the specification for the soil work are in the allowable moisture variation from the optimum and the permissable use of rubber tired rollers. Both specifications allow the soil to be spread in layers 12 inches in thickness prior to compaction for the pervious, and 9 inches prior to compaction for the impervious sections. For the impervious sections, both contracts call for 6 passes of a sheepfoot roller at 250 to 500 p.s.i. but the second contract allows for the substitution of a 4 wheel rubber tired roller with a total weight of 18 to 25 thousand pounds per wheel. The rubber tired roller may also be used on the impervious section of the new job but the first job required, and the second job contractor may elect to use, a 34,000 pound tractor to compact the pervious material.

Constructed under the first contract, the earth dike itself is topped out at an elevation of 85 ft. and is complete with the exception of the paving of the roadway. The switchyard is to be constructed to an elevation 107' or 22 feet higher than the dike and the transition section between the two are to be completed in the present contract.

The height of the water table in this area, artesian flow and the porous condition of the rock all combine, to make the securing of a dry foundation very difficult. The first stage contractor had, in addition to the regular dewatering operation, abnormal flood conditions to combat. His contract called for the curtain grouting operations to be performed from the top of rock after the core trench had been excavated. The grouting was a tremendous help to the contractor in his controlling of the water coming through the exposed rock foundation, and with the help of a well point system in the overburden, he was able to secure a dry bottom.

Under the present or second contract the government is paying the Contractor for constructing cofferdam grout curtains, 17 feet upstream and downstream of the core trench, and extending 20 foot into rock. In addition, the permanent cutoff curtain has been completed prior to the excavation of the core trench. It is expected that with the installation of these three lines of grout curtains, the control of the water during the core trench excavation and the construction of the core in this area will be better. The contractor also proposes to use a well point system to control the water, and deep wells, if necessary, to lower the water table in order to secure a dry foundation for the placement of the soil in the core trench.

In an effort to furnish the field with a minimum percentage of standard Proctor density, shear, consolidation and permeability tests at low densities were run on a composite sample of impervious soil. An angle of internal

friction of 20° was used in the design, but a shear test at density of 90 percent resulted in an angle of 37°. A consolidation of approximately 4 inches was determined to be the practical minimum and since that amount is not appreciably reduced by greatly increasing the density, it was determined that a density of 97 percent would meet this criteria. The design permeability of 1.0×10^{-4} centimeters per second was found to be obtainable at 90 percent of standard density.

Therefore, a 97 percent of standard Proctor density was set as a field minimum. The first contractor elected to continue rolling until this standard was met, and not to hold the government to the 6 passes required by the specifications.

In order to properly control the soils work, a field laboratory equipped to run mechanical and physical analysis, and standard densities was set up. During the construction work, the regular routine control tests were run, including field densities by the sand displacement method. Nine inch cubes of soil were obtained from the finished fill and submitted to our Division Laboratory for check densities, analysis, shear and permeability tests.

The field density tests averaged 101 percent for the impervious material and 103 percent for the pervious material. The undisturbed cubes of impervious material submitted to our Division Laboratory for tests averaged 99 percent of standard density, with a shear angle of 45°, and a permeability figure of 0.23 centimeters per second.

These results indicated that the finished structure was well above the design criteria.

IV. Slope Protection

In order to protect the earthwork from erosion due to the overtopping of the dike, and on the upstream from wave action, both the top, upstream, and downstream slopes are protected by grouted riprap or pavement. The slope protection consists of a 6'' gravel filter blanket placed directly on the compacted fill over which is placed an 18'' thick layer of grouted hand placed riprap.

In preparation for the placing of the filter blanket the fills were cut back to the desired slopes, then the gravel was placed by trucks from the top and bottom. The filter material was well graded gravel with a maximum size of 1-1/2 inch.

After the filter material was in place the maximum 200 pound riprap was hand placed so that in general the required 18 inch thickness consisted of one piece of stone in depth.

The grouting of the riprap was done in two courses. The grout was mixed in a regular concrete mixer and transported to the finished riprap surface in 1 cubic yard concrete buckets. The first course was a 2 parts sand, 1 part cement and 8-1/2 parts water. This mix was dumped on the surface of the riprap and hand broomed into the voids, the next course, a dryer mix; 2 parts sand, 1 part cement and 7-3/4 parts of water, was then dumped on the riprap surface and broomed so as to completely fill the voids.

V. Grouting

Our office believes that the grouting at Jim Woodruff Dam has produced some interesting problems and experiences and that their solution has resulted in the development of new techniques and procedures.

In the first contract, the materials used in grouting the curtain consisted of cement and water while in the second contract, in order to grout large voids at less expense, the neat grout is being supplemented by a mix consisting of sand - flyash, fluidifier, cement and water.

As a general procedure drilling and grouting for all of the curtain has been divided into three zones of depth. Each zone could be divided into two or more stages, if during the drilling operations there was a loss of drill water, a cavity was discovered or artesian flow developed. When any such condition was encountered, the drilling operations were stopped and the hole grouted to refusal, before the drilling was resumed to the remaining depth of that zone. It is the opinion of personnel of our office that for our foundation rock this zone and stage method has produced an excellent grouting job and one that will perform the design function.

In general, the rule of one pound per square inch for every foot of overburden for washing and grouting pressure was used for all operations. One exception to the rule was made in the first contract work where the grouting was done after the overburden had been removed, then the pressures were lowered due to the possibility of breakouts both in the rock and around the casings. The other exception was when the grouting was done through the completed earth structure where the surcharge of the dike was taken into consideration and the pressures slightly increased. All of the grouting for the curtain has been done by the split spacing method. This method involves the drilling and grouting of zones of holes on 20 foot centers, then progressively reducing the spacing by drilling and grouting the zones of intermediate holes until the desired 5 foot spacing is obtained. All zones of holes are grouted until satisfactory refusal is obtained.

All holes prior to grouting are rod washed and pressure tested. The consumption rate during pressure testing furnishes an indication of the possible grout takes and the mix that may be used while the washing indicates any connections with other holes, possible breakout areas and helps wash out the hole.

The curtain for the majority of the first contract was grouted after the core trench had been excavated. This method proved relatively unsuccessful due to the difficulty in setting the grout risers into rock, keeping them seated during drilling and grouting operations and the frequent breakouts in the exposed rock. The grout mixture used ranged from a 1 cement to 4 water to a 1 cement to 0.6 water, however, the majority of the grout pumped was either a 1 to 1 or dryer.

Due to the difficulties and the cost involved in the large quantity of neat cement grout pumped, the grout curtain in the first stage work was not completed but was set up as an item of work in the second contract. In the first contract, for 1500 horizontal feet of grout curtain, a total of about 18,000 lineal feet of holes were drilled with 148,370 cubic feet of grout pumped at a cost of $248,000. This averages 8.1 cubic feet of consumption per foot of hole and the cost of the 50 foot depth of curtain per horizontal foot of dike was $195.00.

The curtain grouting for the second stage work includes the installation of a curtain under the switchyard and transition section and the completion of the curtain under a portion of the earth dike completed in the first contract.

Using sanded grout in the switchyard and transition section for 520 linear feet of grout curtain, a total of about 7,955 lineal feet of hole were drilled and 152,450 cubic feet of grout pumped at a cost of $205,800. This averages 19 cubic feet of consumption per foot of hole and the cost of the 50 foot depth of curtain per horizontal foot of dike was $367.00.

As for a comparison of costs of the two grouts, the cost per cubic foot was $1.72 on the first contract and $1.35 for the second. Taking into consideration the rise in prices over the five year period, this represents a considerable savings.

All the grouting is being done through 2-1/2" ID casings seated into rock. When working through overburden or earth fill a 3" hole is drilled three feet into rock, the casing is set and the hole washed clean, then about a half cubic foot of grout is poured into the casing, the casing lifted a few inches and then driven back into the drilled hole.

All curtain holes have been drilled by pneumatic air drills equipped with AX rods, and job made carbolloy bits which averaged about 1-3/8" in size.

The neat grout mixes used in the second contract vary from a 1 cement - 0.8 water to a 1 cement - 1.5 water. The sanded mixes vary from 1 cement - 1 flyash - 1 sand - 1% fluidifier - 1.3 water to a 1 cement - 1 flyash - 2% fluidifier - 6 sand - 3.3 water.

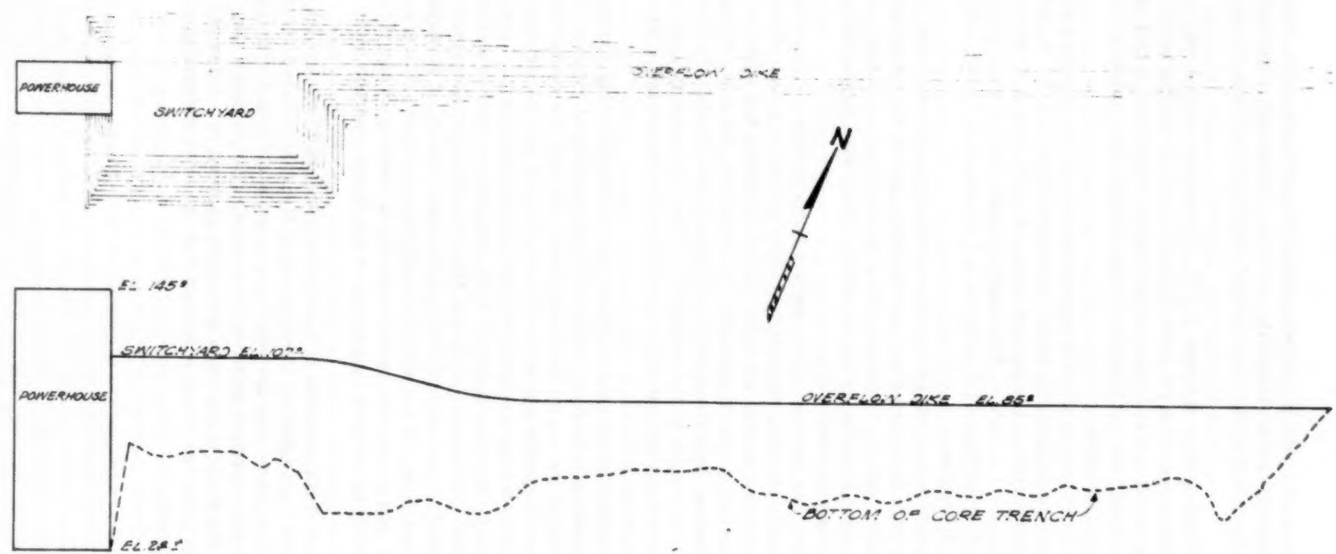
The grouting procedure was usually to start with about 30 cubic feet of a wet neat mix to lubricate the grout lines and the hole, and if the consumption and pressures are satisfactory, to follow with a thicker neat or sanded grout mix. If the hole continued to take grout the sand content was increased until the 6 parts of sand mix was reached.

If a cavernous area was being grouted and the consumption continued with no indication of refusal, grouting was stopped after a total of about 1000 cubic feet of grout had been pumped into the hole. After the grout had taken its final set, grouting operations were resumed. If, however, the hole showed any indication of refusing, operations were continued even though the total quantity pumped was in excess of the allowable. This intermittent grouting it is believed minimized the spread of the grout thus saving money while still securing a satisfactory curtain.

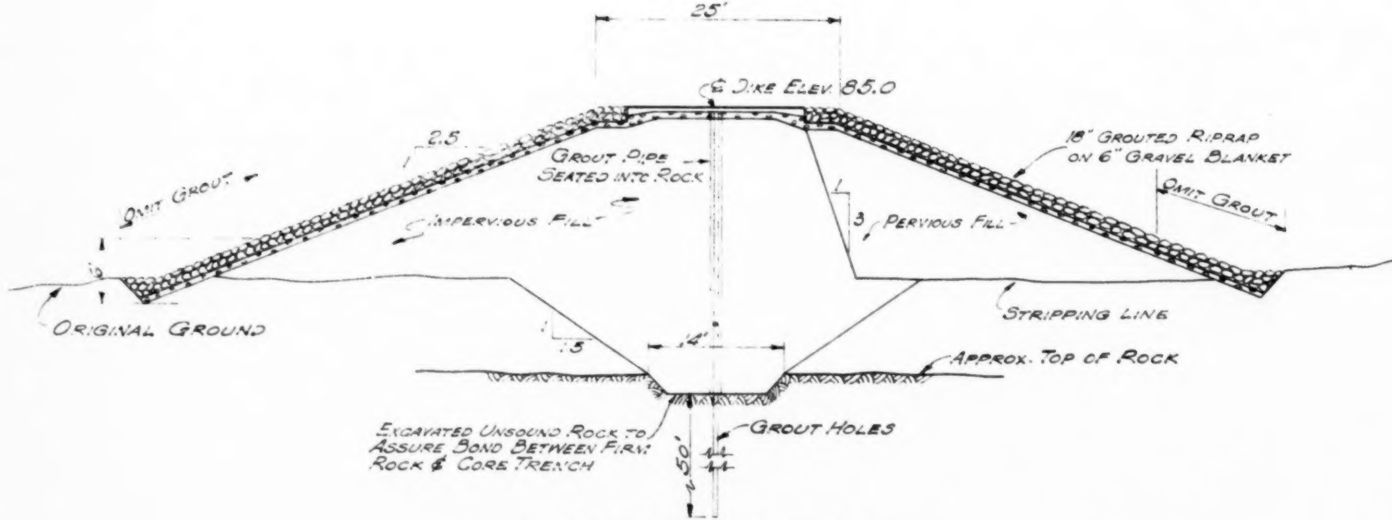
In our curtain grouting operations we have used three different approaches to the problem of securing a good job, first after excavating of the core trench; second before excavating the core trench and third, through the completed dike and core. Add to these the use of two types of grout and the description is complete. As of date we have no positive method of comparison; the grouting from the excavated core trench seems to be the poorer method and through the completed dike the better, assuming that the grouting operations are consistent. For aid in water control in drying up the core trench, it appears best to grout prior to excavation. In grouting through the completed dike chances are taken that water and grout may cause breaking or heaving of the structure.

It will be interesting to observe and compare the three sections after the reservoir is filled.

This office feels that the grouting at this project will perform the job it is designed to do. In the excavation of other areas of the dam grouted sections have been exposed, that show that the voids are completely filled with the sanded grout. Test holes drilled between the regular curtain holes have shown by their cores that the voids in the rock are being grouted up. Attempts to inject grout in these holes have also indicated that the area surrounding the test hole is well grouted.



EARTH FILL OVERFLOW DIKE
JIM WOODRUFF DAM
Fig. 1



TYPICAL SECTION
EARTH FILL OVERFLOW DIKE
JIM WOODRUFF DAM
Fig. 2

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